EVALUATION OF OVERTURNING FORCES ON SHEAR WALL BUILDINGS

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ABSTRACT

The strong motion instrumentation program (SMIP) of the California Division of Mines & Geology (CDMG) has been designed to instrument specific building types in specific areas of California where strong ground motion records may be readily obtained from active seismic sources. The records are intended for use by structural engineers and researchers in developing analytical and design procedures that more accurately represent the building's behavior in earthquakes. Recent California earthquakes have provided significant data on a series of instrumented shear wall buildings along with observable data on their earthquake performance.

This paper presents a detailed investigation of three high-rise shear wall buildings in the nine- to ten-story range with three different shear wall configurations: perimeter walls, core walls and distributed walls. The dynamic earthquake response of these buildings is assessed to evaluate overturning forces in the shear walls under three recent northern California earthquakes: 1984 Morgan Hill; 1986 Mt. Lewis; and 1989 Loma Prieta. Two methods of data reduction and analysis are employed in the investigation to assess the significance of soil-structure interaction on building overturning forces. These include: simplified data analysis procedures using recorded motions, mode shapes and building weights to assess dynamic performance and three-dimensional linear elastic dynamic analyses using soil-structure models for the shear walls and foundation systems.

Realistic three-dimensional models of the structures refined through system identification techniques are used to study the response to the three earthquakes. These analyses indicated that under the larger earthquakes structural softening occurred that was associated both with soil strain levels as well as shear wall cracking. The analytical results are compared with code procedures for predicting the periods of the structures as well as the distribution of overturning forces.

INTRODUCTION

The interaction between shear wall structures and their foundation system under lateral loads that induce overturning forces in the walls, has normally been neglected in conventional building code seismic design practice. Prior to the SMIP program of instrumentation, shear wall structures were not instrumented to record the rocking component of motion induced by earthquake. It has only been within the last ten years that adequate instrumentation of shear wall structures along with the occurrence of earthquakes in the vicinity of the instrumented buildings has provided data for use in analytical investigation and verification of design procedures for seismic overturning.

The scope of this investigation was to evaluate the overturning forces in three separate buildings with three different types of shear wall systems, two types of foundation systems, and recorded motions from three separate events in two of the buildings and one common event in all three buildings. Three-dimensional linear elastic soil-structure models were formulated using conventional computer codes (SAP90 and ETABS) to evaluate the influence of overturning forces in the shear walls of the buildings. System identification techniques along with conventional structural modeling procedures were used to define and refine the models and to identify the contributions from soil and structural deformation in the recorded response. Recorded motions at the base of the structures were input to the models and computed at the roof for comparison with the recorded motions as a confirmation of the basic structural model. The computed forces in the walls of the structure were investigated in terms of overturning effects.

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The buildings, their California Strong Motion Instrumentation Program (CSMIP) number, location, shear wall type, and recorded earthquakes are listed in Table 1.

TABLE 1 - BUILDING CHARACTERISTICS AND RECORDED EARTHQUAKES

Building Identification		Description of Structure	Recorded Earthquakes
Bldg. 1	CSMIP Station 57355 San Jose, CA	10-story commercial building perimeter shear walls and frames mat foundation	1984 Morgan Hill 1986 Mt. Lewis 1989 Loma Prieta
Bldg. 2	CSMIP Station 57356 San Jose, CA	10-story residential building distributed bearing/ shear walls pile foundation	1984 Morgan Hill 1986 Mt. Lewis 1989 Loma Prieta
Bldg. 3	CSMIP Station 58394 San Bruno, CA	nine-story government office building central core walls mat foundation	1989 Loma Prieta

Buildings 1 and 2, are located about a 1/4 mile apart (three city blocks) in the central part of San Jose. As shown in Figure 1, the San Jose buildings are located about 18 miles north of the epicenter for the Loma Prieta earthquake, 12 miles east of the epicenter for the Morgan Hill earthquake, and 15 miles southwest of the Mt. Lewis earthquake epicenter. Building 3 in San Bruno is located about 56 miles north by northwest of the Loma Prieta epicenter.

DESCRIPTION OF BUILDINGS

No. 1 - Ten-story Commercial Building - San Jose

The ten-story commercial office building shown in Figure 2 has plan dimensions of 190 feet x 82 feet. It's total height above the ground surface is 124 feet and it has a single basement that extends 17 feet below ground. The building is constructed of reinforced concrete with exterior shear walls and interior moment frames as the lateral force resisting system in the transverse (east-west) direction. In the longitudinal (north-south) direction the lateral loads are resisted by two exterior and two interior reinforced concrete moment frames. The floor and roof diaphragms are composed of a one-way slab and joist system. The building's foundation consists of a five foot thick reinforced concrete mat. All of the reinforced concrete elements of the building above ground floor level are constructed of light weight concrete. The remaining grade level and below construction consists of normal weight (hard rock) concrete. Although the building was designed in 1965 in accordance with the provisions of the 1964 UBC, careful consideration was given to detailing the moment frames so as to ensure ductile behavior during earthquakes.

The CDMG strong motion instrumentation of Building 1 is shown in Figure 2. Thirteen accelerometers have been installed on the building to record both translational and torsional effects in the floor slabs as well as vertical rocking motions in one of the shear walls.

No. 2 - Ten-story Residential Building - San Jose

The ten-story residential building in San Jose shown in Figure 3 is a reinforced concrete structure with one-way post-tensioned floor slabs. The overall plan dimension of the building is 209.5 feet by 63.5 feet. The height of the building above ground surface is 94 feet. The building's transverse (east-west) wall system is designed as a combination bearing wall for vertical loads and shear wall for lateral loads. In the longitudinal direction (north-south) the lateral force resisting system consists of a series of intermittently spaced shear walls along both sides of the interior central corridor. Two of the longitudinal interior shear walls are terminated at the sixth floor level. The lateral stiffness in the longitudinal direction is significantly lower than the stiffness in the transverse direction. The building is founded at grade level on precast-prestressed concrete piles that are placed directly below the bearing/shear walls. All of the shear walls in the building are constructed of normal weight (hard rock) concrete while the elevated post-tensioned floor slabs are constructed of light weight concrete. The piles, pile caps, and grade slab are all constructed of normal weight concrete.

The building was designed in 1970 and 1971 according to the provisions of the 1970 UBC. The 1971 San Fernando earthquake occurred while the structure was under design. The designers made a point of incorporating lessons learned from the performance of similar shear wall structures in the San Fernando earthquake. Additional steel was added to the shear walls of the building beyond that called for in the 1970 UBC to enhance the building's earthquake resistance. Construction of Building 2 was completed in 1972.

A total of 13 strong motion accelerometers have been deployed in Building 2 under the CDMG Strong Motion Instrumentation Program. This array of accelerometers is designed to measure horizontal building motions at the roof, sixth floor and ground floor as well as vertical building motions, rocking of a major transverse shear wall, and transverse flexural deformations in the long narrow roof diaphragm.

No. 3 - Nine-story Government Office Building - San Bruno

Figure 4 illustrates the nine-story government office building in San Bruno, California. The building's plan dimensions are 192 feet by 84 feet and its roof height is 104 feet above the first floor. The structural lateral load resisting system consists of a central elevator/stair core constructed of reinforced concrete. The vertical load carrying system consists of post-tensioned flat slabs without drop panels that are supported on concrete columns. The foundation system consists of a ribbed mat composed of a thick floor slab supported on deep reinforced concrete grade beams. The building was constructed in 1972.

The CDMG strong motion instrumentation of the building consists of 16 accelerometers as shown in Figure 4. The accelerometer locations have been chosen to provide data on the flexibility of the roof diaphragm, lateral transnational performance of the building, and rocking of the core wall in the transverse (north-south) direction.

SOIL PROPERTIES

Soil reports for the design of Buildings 1 and 2 in San Jose were located through the assistance of the design engineers. These provided information on the soil profiles and in the case of Building 2 the relative stiffness of each soil layer based on actual sampling blow counts. From this information the shear modulus, poissons ratio and density of the soil materials were estimated at in situ strain conditions as well as large strain conditions associated with earthquake. These properties were used in modeling the soil foundation stiffness characteristics under the mat and pile foundation for Buildings 1 and 2, respectively.

For Building 3, no soil report was obtainable. Instead, system identification techniques were employed along with best estimate soil properties for the area to arrive at an approximate soil-spring stiffness for building rocking.

RECORDED MOTIONS

Strong motion records from each of the three buildings have been published by CDMG along with fourier spectra and response spectra. Table 2 summarizes the recorded peak accelerations in each of the three buildings for the various earthquakes. It is evident from this table that the two buildings in San Jose experienced significantly higher levels of ground shaking from the Loma Prieta earthquake than either the Morgan Hill or Mt. Lewis events.

TABLE 2 - SUMMARY OF RECORDED PEAK ACCELERATIONS IN G's

Bldg.	Earthquake	Longitudinal Direction		Transverse Direction	
		Base	Roof	Base	Roof
1	1984 Morgan Hill	.058	.180	.061	.220
	1986 Mt. Lewis	.029	.078	.034	.077
	1989 Loma Prieta	.092	.254	.105	.375
2	1984 Morgan Hill	0.056	0.216	0.061	0.133
	1986 Mt. Lewis	0.030	0.119	0.036	0.082
	1989 Loma Prieta	0.094	0.371	0.127	0.242
3	1989 Loma Prieta	0.114	0.320	0.158	0.372

INTERPRETATION OF BUILDING RESPONSE FROM RECORDED MOTIONS

Strong motion records for the buildings have been analyzed using the raw data provided in digital form by CDMG to assess the mode shapes, periods of vibration, damping, base shears and overturning moments. These findings have been corroborated by previous studies. Table 3 summarizes the fundamental translational or torsional modes of vibration for each of the three buildings and for each of the three earthquakes recorded in Buildings 1 and 2 in San Jose.

TABLE 3
PERIODS IN SECONDS

MH - Morgan Hill
ML - Mt. Lewis
LP - Loma Prieta

Building 1

Bldg. No.	Mode	Periods from Recorded Motions: MH (1984)	ML (1986)	LP (1989)	Computer Model
1	1	0.91 (N-S)	0.91 (N-S)	1.01 (N-S)	1.03 (N-S)
	2	0.61 (E-W)	0.61 (E-W)	0.75 (E-W)	0.77 (E-W)
	3	0.37 (TOR)	0.39 (TOR)	0.44 (TOR)	0.44 (TOR)
2	1	0.65 (N-S)	0.63 (N-S)	0.73 (N-S)	0.59 (N-S)
	2	0.43 (E-W)	0.41 (E-W)	0.43 (E-W)	0.43 (E-W)
	3*	--	--	--	0.38 (TOR)
	4	0.18 (N-S)	0.18 (N-S)	0.19 (N-S)	0.17 (N-S)
3	1	--	--	1.45 (N-S)	1.80 (N-S)
	2	--	--	1.38 (TOR)	1.42 (TOR)
	3	--	--	1.00 (E-W)	1.15 (E-W)

* Mode 3 is a torsional mode in the computer model at T = 0.38 seconds.

It is evident from Table 3 that the fundamental period of vibration during the Loma Prieta earthquake is longer for the two translation and torsional components of motion in Building 1 and the longitudinal (north-south) component of motion in Building 2. This is due in part to the strain dependent softening of the supporting foundations and in part to the cracking and softening of the shear walls under the larger motions produced by the Loma Prieta event.

COMPUTER MODELS OF BUILDINGS

Figures 5, 6 and 7 present the three-dimensional elastic finite element models for Buildings 1, 2 and 3, respectively. All three buildings were found to have rigid floor diaphgrams based on the recorded accelerations and displacements. Thus Buildings 1 and 2 have been modeled using the ETABS program and Building 3 was modeled using the SAP90 computer program. An additional level has been added to the models to incorporate soil rocking stiffness characteristics. The models for Buildings 2 and 3 represent fixed base solutions with structural properties based on gross section properties.

ANALYTICAL RESULTS

Periods and Mode Shapes of Vibrations

Table 3 summarizes the computed frequencies for each of the three models shown in Figures 5, 6, and 7. Mode shapes associated with these periods of vibration are shown in the same figures as the models.

The periods computed for Building 1 are from the computer model developed specifically to approximate the conditions with the Loma Prieta earthquake. This model includes soil springs to simulate rocking at the base. Also, cracked section properties were assumed for the moment frames at 80 percent of gross section properties and for the shear walls at 50 percent of the gross section properties as suggested by Wallace, et.al. (1990). This reduction in structural stiffness produced excellent correlations with the Loma Prieta records for the north-south and east-west directions.

Time History Analyses

Figure 8 presents the plot of time history motion recorded parallel to the transverse (east-west) shear walls of Building 1 as recorded at the roof level and as computed with the analytical model for the Loma Prieta earthquake. The solid line represents the recorded earthquake and the dotted line represents the computed response using as input motion the record from the base of the structure. In general the correlation between computed and recorded roof motions is reasonably close.

The lower half of Figure 8 represents the response spectra computed from the recorded motions at the roof versus the response spectra of computed motions using the analytical model shown in Figure 5. The response spectra comparison also shows good correlation.

Figure 9 presents the comparison of roof motions for Building 2 under the Morgan Hill earthquake recorded in the longitudinal and transverse directions versus the computed response. The correlation in this case is good considering the model has a fixed base.

CONCLUSIONS

The following are conclusions from the investigation of the three shear wall buildings:

1. The earthquake records provided ample evidence that foundation/shear wall rocking occurs under small to moderate size earthquakes in these structures.

2. Lengthening in building period occurs both due to structural rocking and due to inelastic behavior (cracking) of the shear walls.

3. The records provided ample evidence that the reinforced and prestressed concrete floor diaphragms in all three buildings were rigid relative to other building deformations.

4. Rocking of the shear wall/foundation system can be modeled with the appropriate soil properties or through system identification using the records in the building as a basis for correlation.

5. The geometry of Building 2 resulted in more walls in the transverse direction than the longitudinal direction. This may explain why no cracking was reported or observed in the transverse walls while the longitudinal walls were reported to have cracked under the Loma Prieta earthquake. It also explains why the building is stiffer and has shorter periods in the transverse direction than the longitudinal direction.

6. In the transverse direction of Building 1 and the longitudinal direction of Building 2 lengthening of structural periods under the Loma Prieta earthquake relative to the periods computed under the Morgan Hill and Mt. Lewis events was probably due to cracking in the walls.

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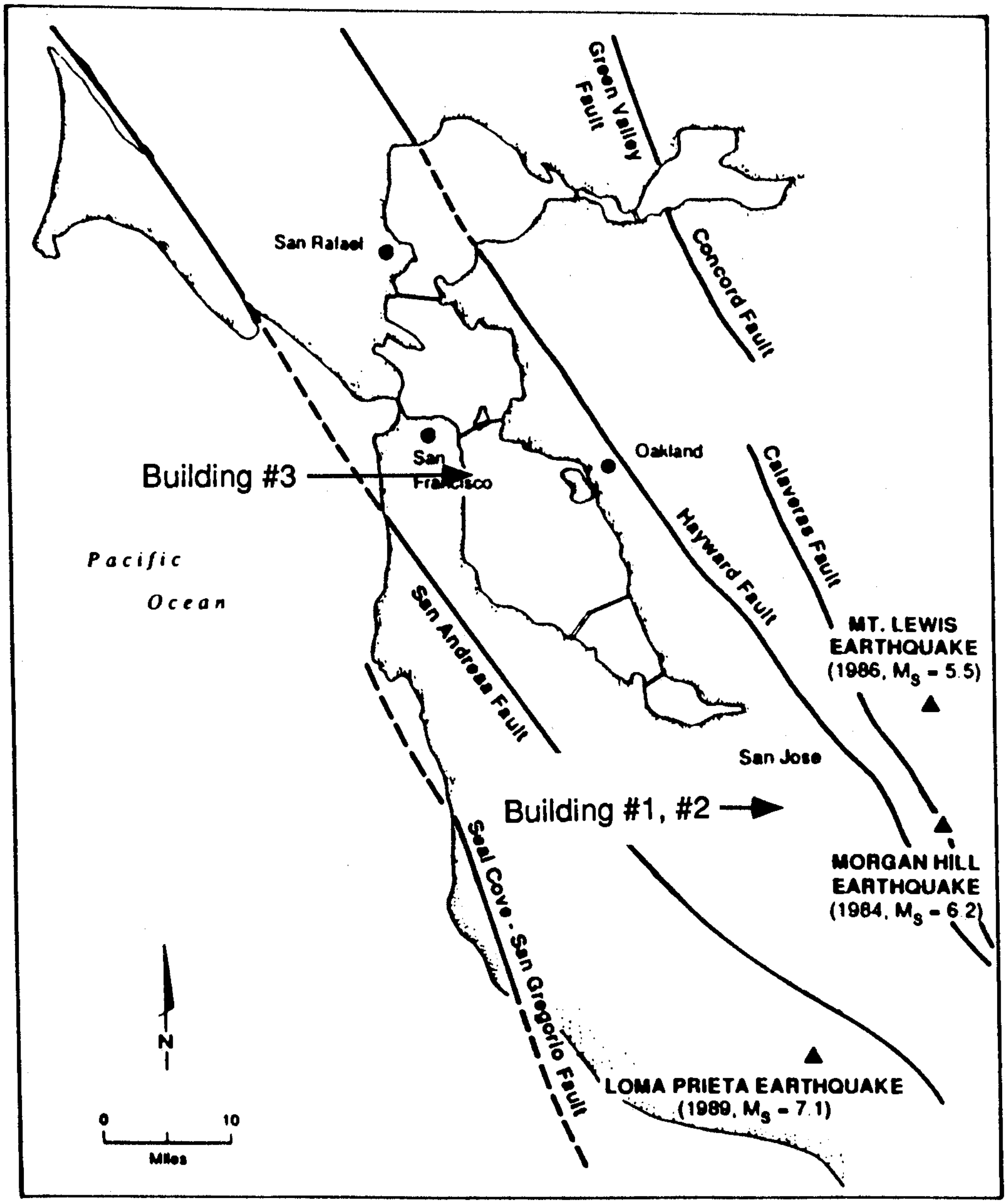


Figure 1. Proximity of Major Faults and Earthquake Epicenters to Building Locations

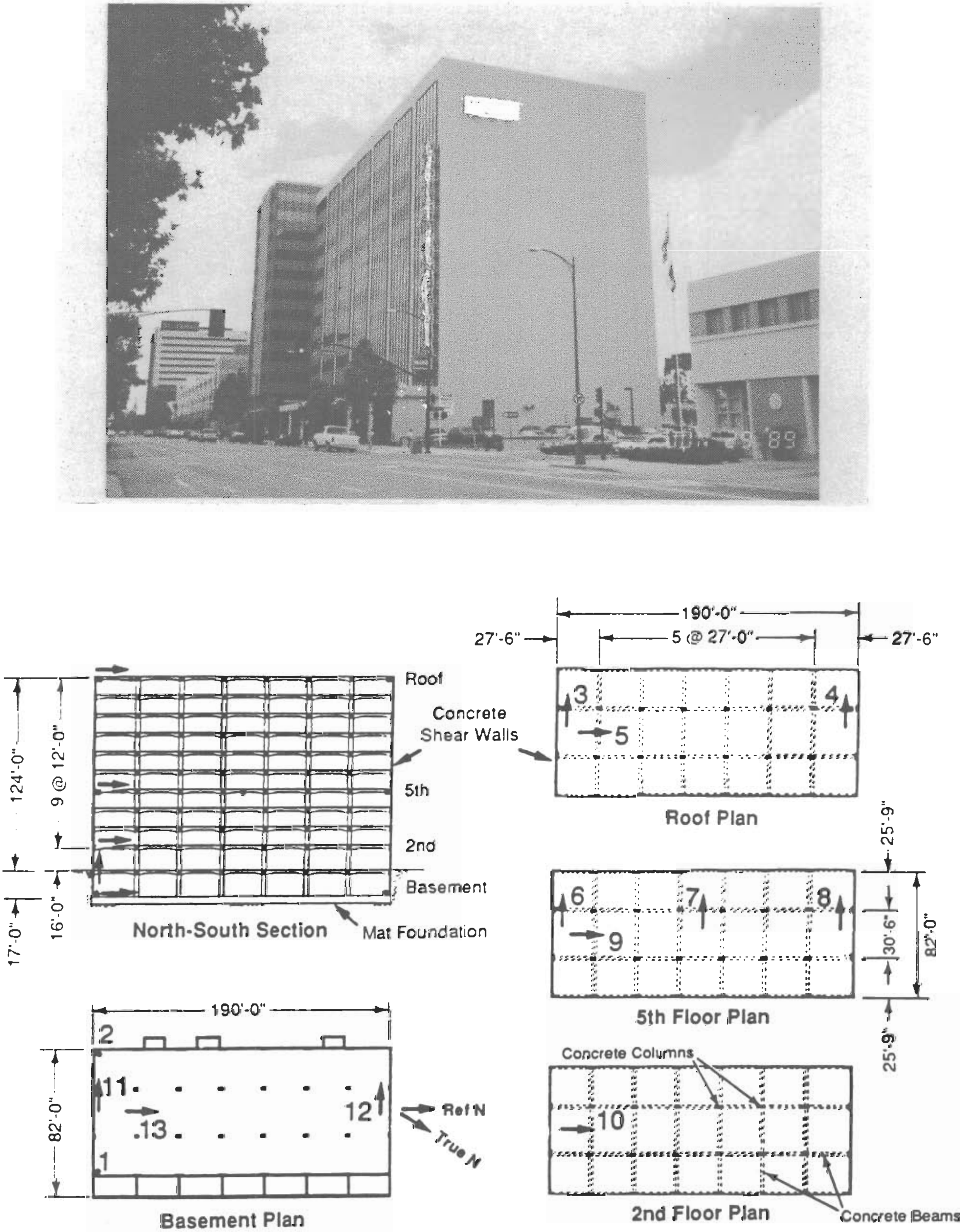


Figure 2. Building #1 - 10 Story Commercial Building, San Jose

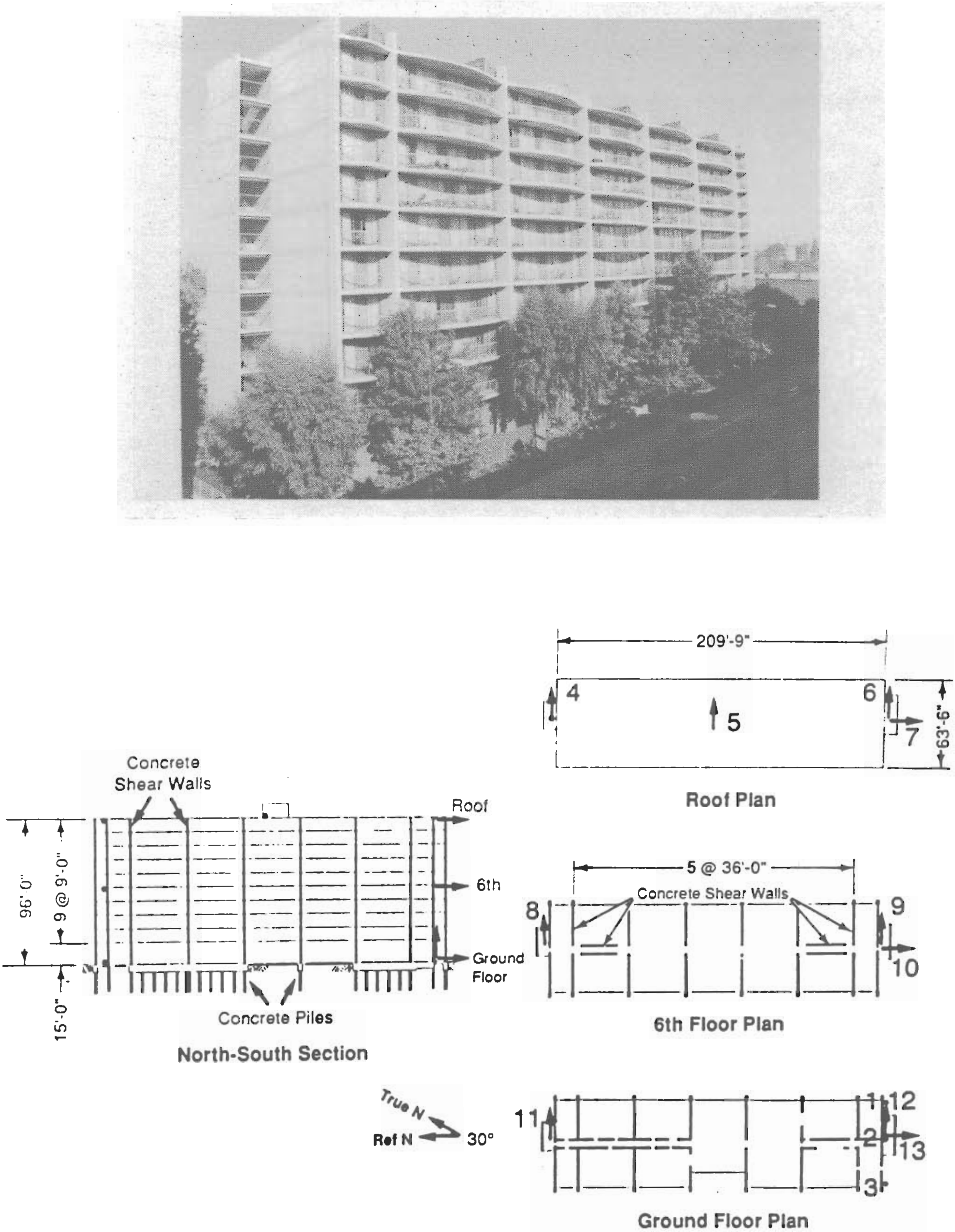


Figure 3. Building #2 - 10 Story Residential Building, San Jose

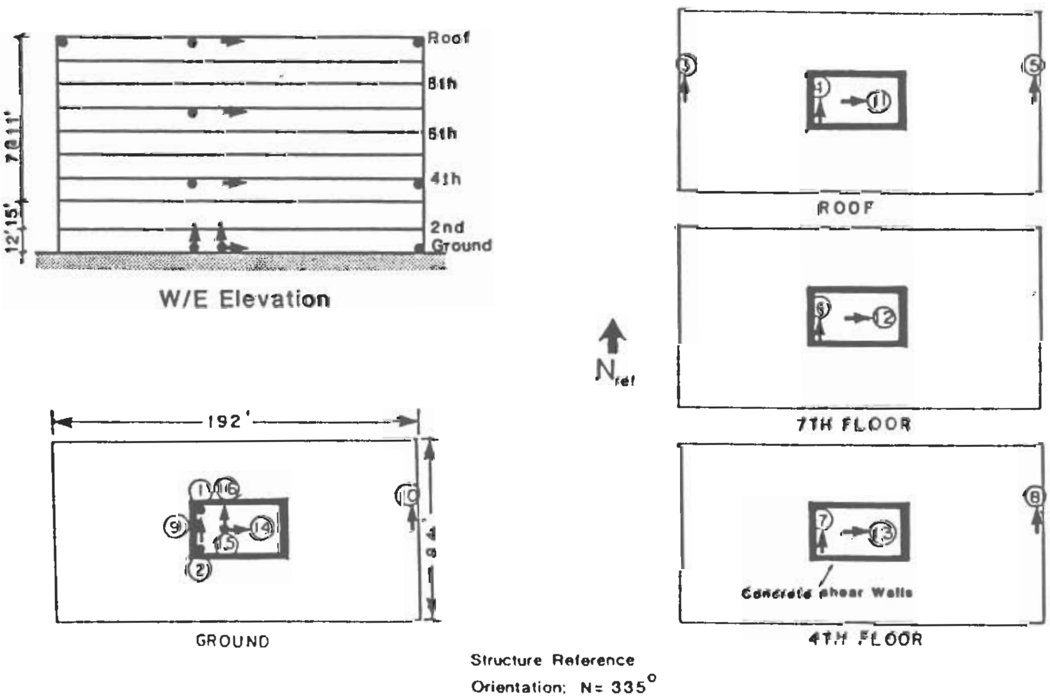
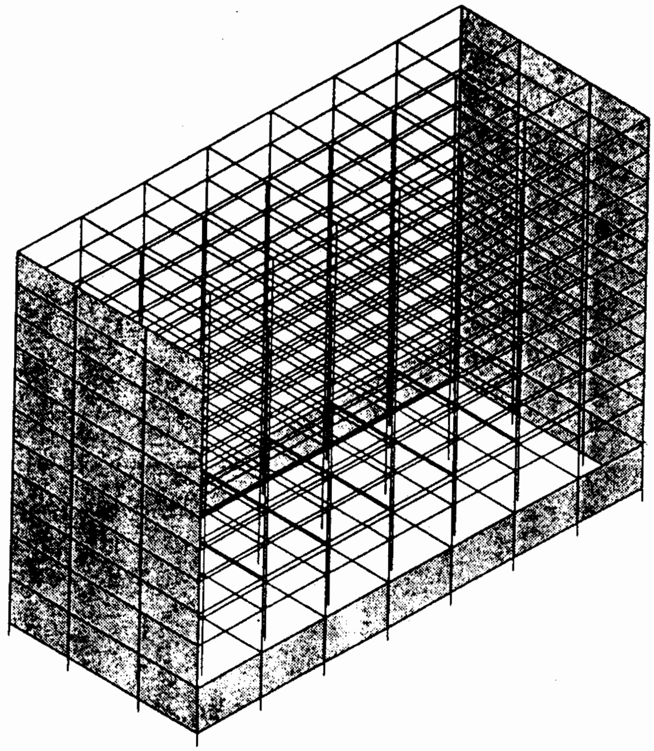
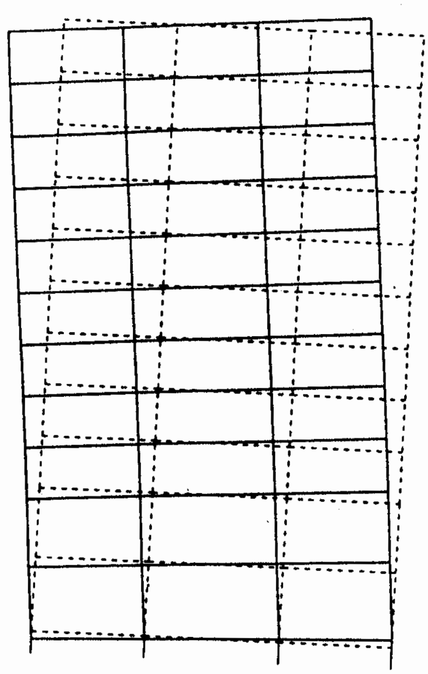


Figure 4. Building #3 - 9 Story Government Office Building, San Bruno



ETABS COMPUTER MODEL



FIRST MODE SHAPE - TRANSVERSE DIRECTION

Figure 5. Building #1 - Model and Mode Shape

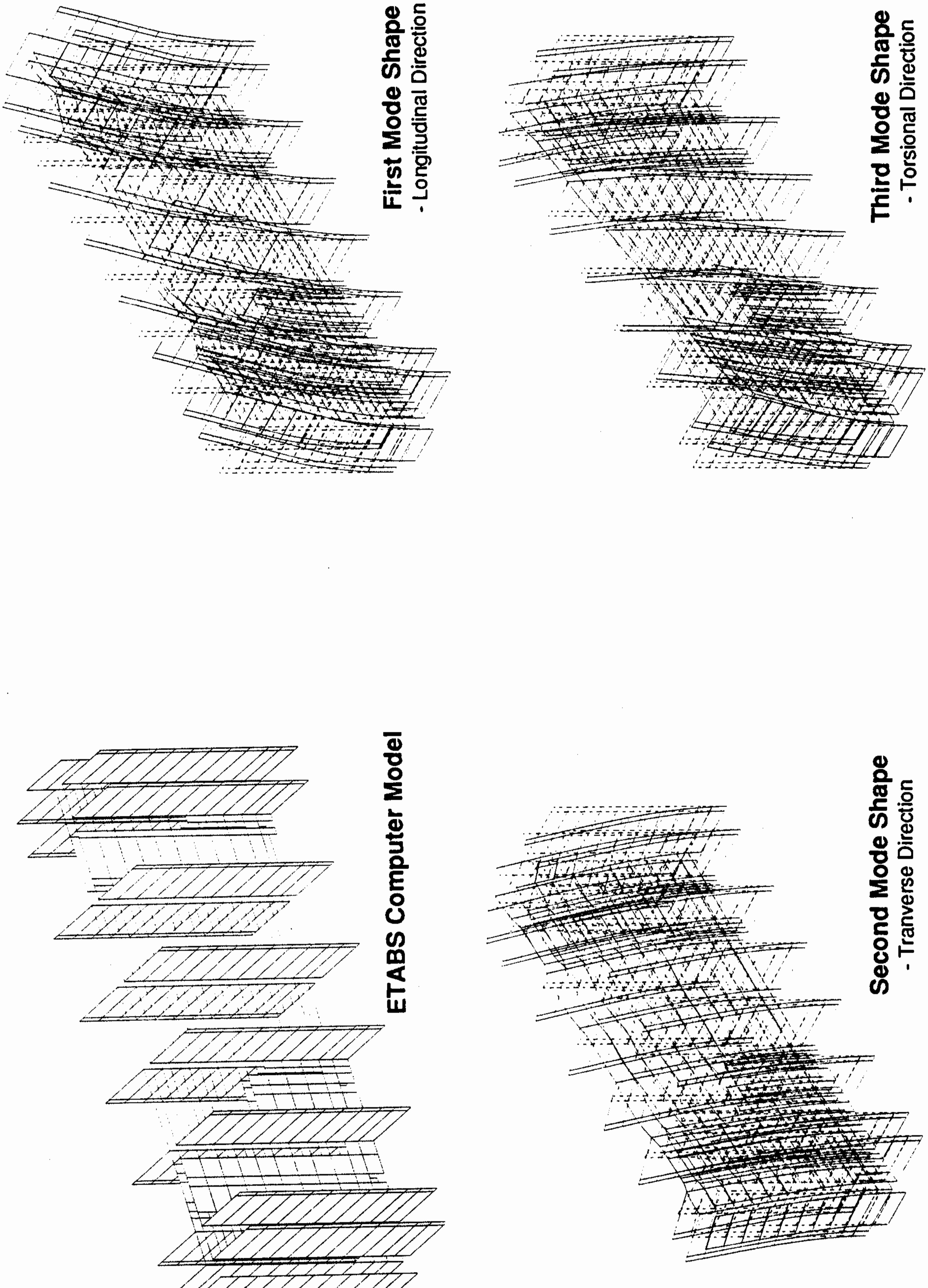


Figure 6. Building #2 - Model and Mode Shapes

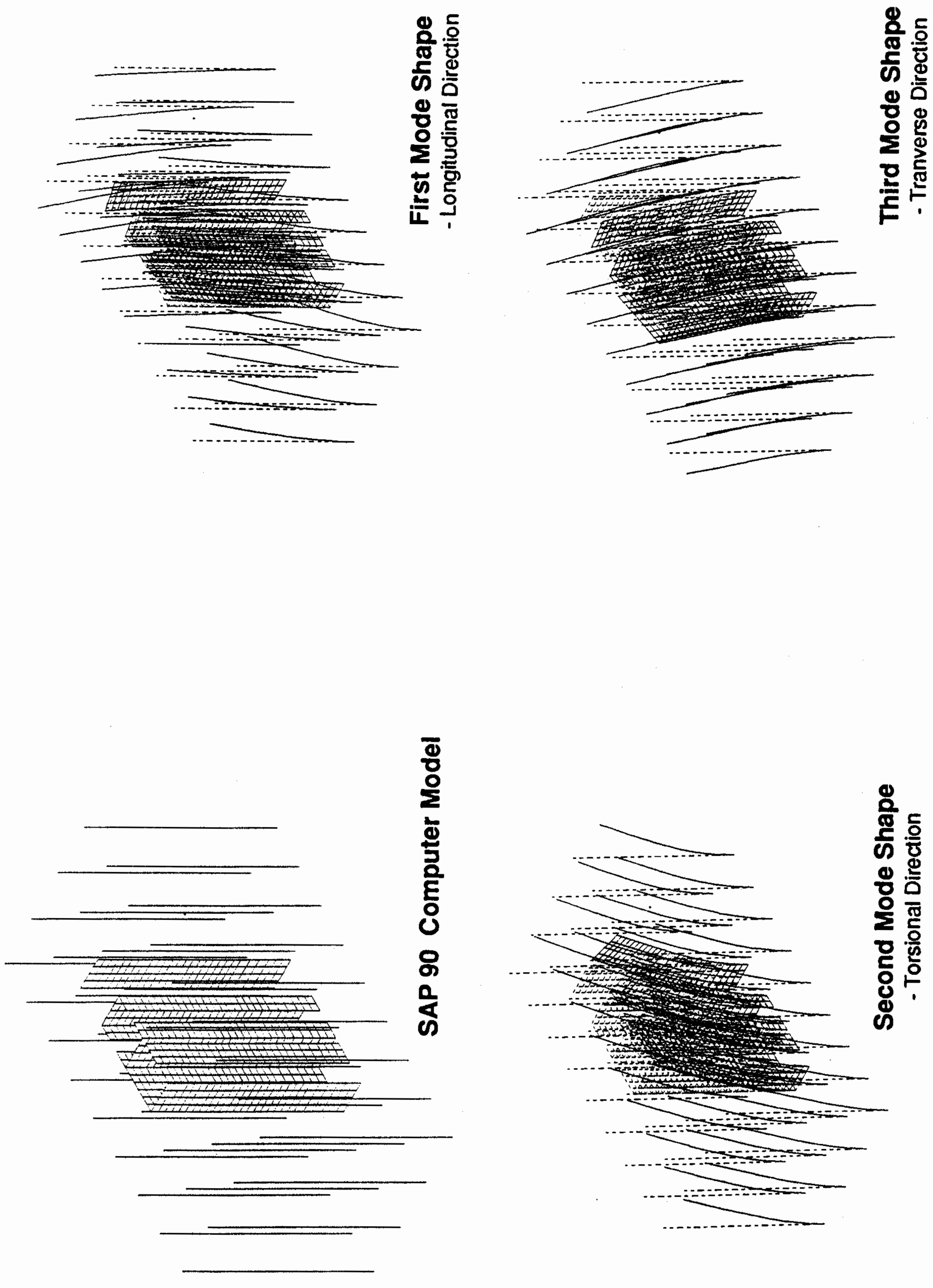
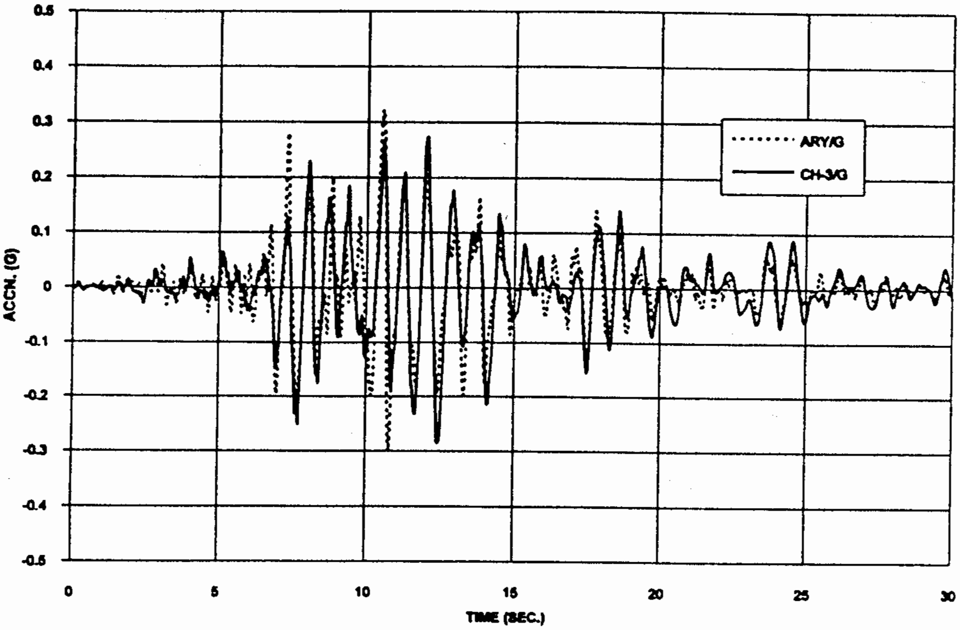
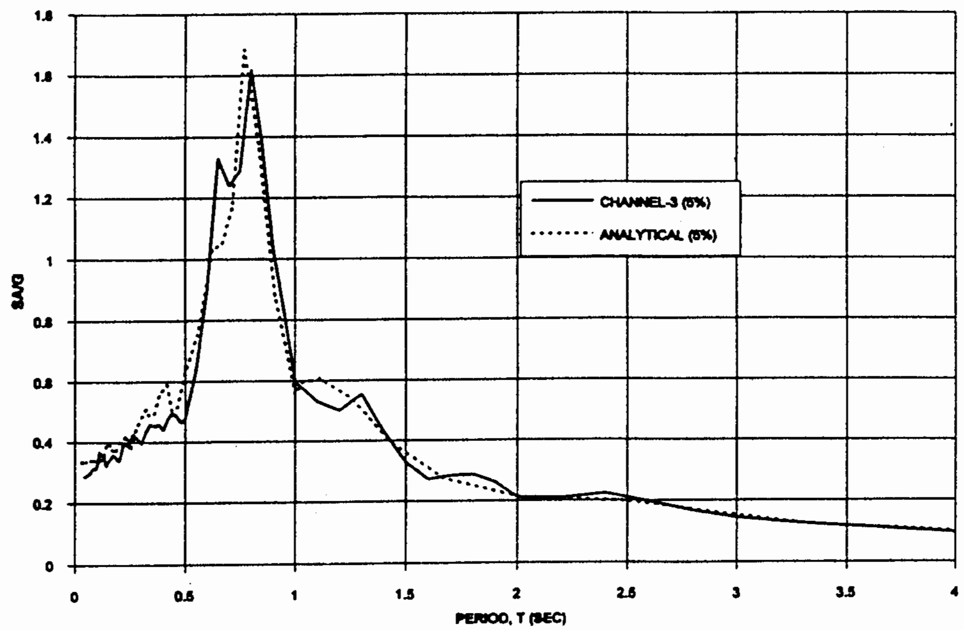


Figure 7. Building #3 - Model and Mode Shapes

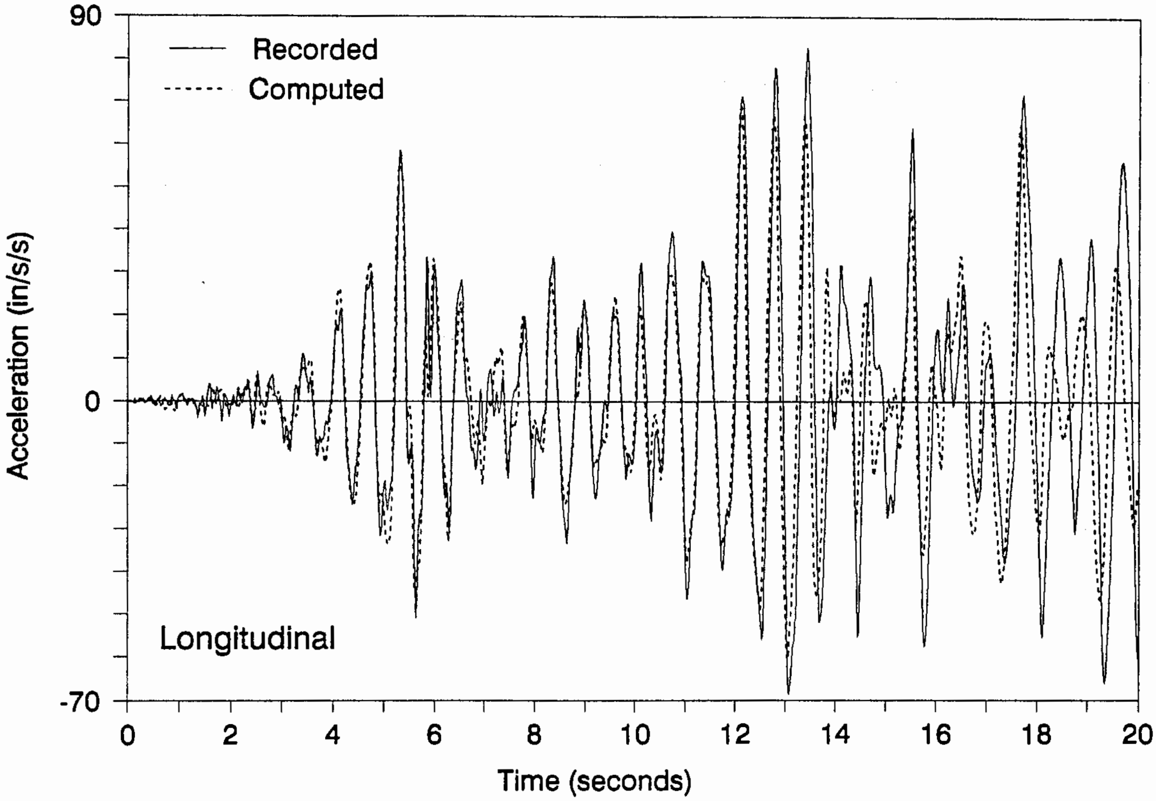


TIME-HISTORY OF RECORDED AND COMPUTED ROOF ACCELERATIONS - TRANSVERSE DIRECTION



RESPONSE SPECTRA OF RECORDED AND COMPUTED ROOF ACCELERATIONS - TRANSVERSE DIRECTION

Figure 8. Building #1 - Computed and Recorded Roof Motion Response Loma Prieta Earthquake



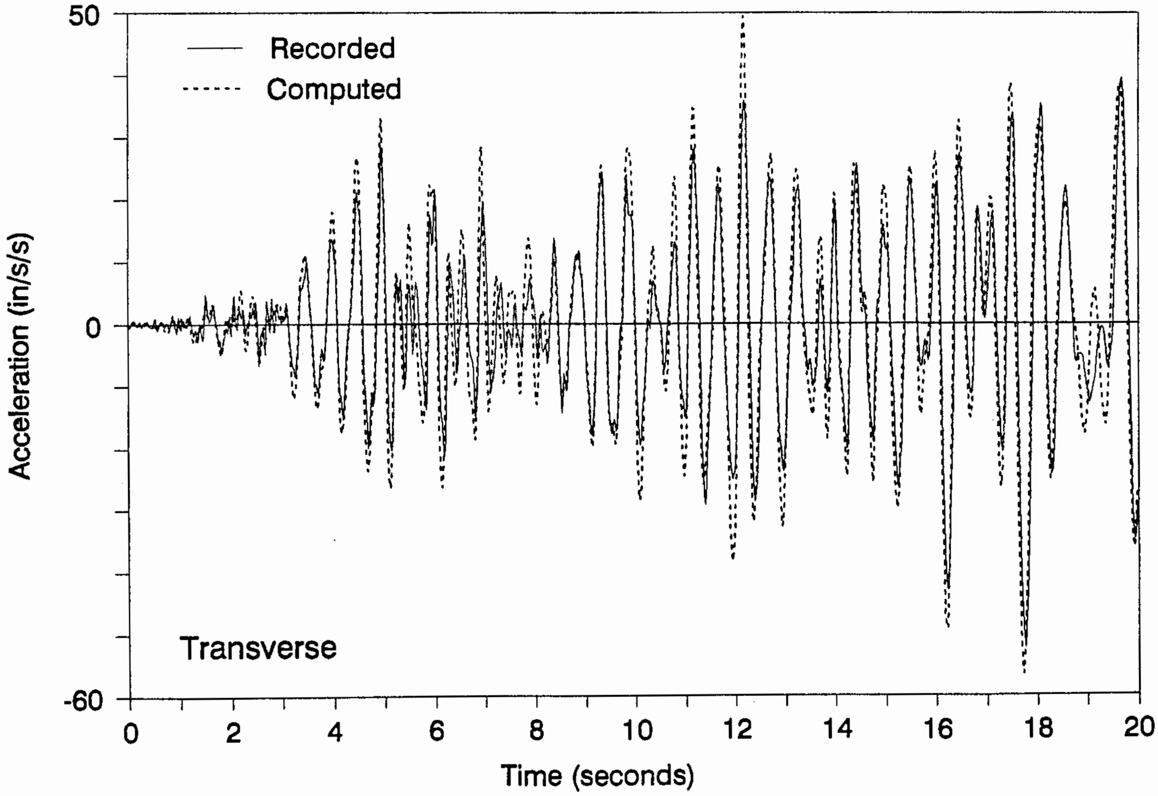


Figure 9. Building #2 - Computed and Recorded Roof Acceleration
Morgan Hill Earthquake